Current and historical monitoring data at the Kiirunavaara footwall

A review

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Abstract
This literature review covers the current and historical surface and underground monitoring systems. Available data from both active and decommissioned systems is collected and presented. The review also covers data retrieved through temporary measurements such as core-drilling, geological mapping and damage mapping. Geotechnical data is sorted by origin and type categorised as rock mass properties, joint set data or equivalent input data for numerical modelling. Measurement data is categorized by origin, observations, field measurements and laboratory data. The results from these geotechnical studies are summarized and referenced to the original publication and put into context to a large scale footwall failure.

The study is concluded by a review of some of the previous numerical models covering the Kiirunavaara footwall. The review has found that during the last decade the numerical analyses related to the footwall have all used the same sets of data, all originally compiled and presented in 2001 and that the footwall rock mass has consistently been treated as a single coherent geological unit. Mohr-Coulomb failure models have been used throughout.
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1. Introduction

As of the last 30 years the Kiirunavaara mine has experienced a slow but progressive fracturing and movement in the footwall rock mass. The footwall instability is directly related to the LKAB utilisation of the SLC (Sublevel caving) method. This is a cost-effective mass-mining method allowing a high level of automation but inherently causes the host rock, particularly on the hangingwall side, to fail progressively and to fill the void caused by the ore extraction. This progressive caving is a prerequisite for optimal performance of the operation. As mining progresses and the orebody is removed the footwall contact becomes de-stressed and assumes a slope-like geometry. The footwall “slope” is partially supported by the caved rock masses from the hangingwall (Villegas & Nordlund, 2008). Despite this, damage both on the footwall crest and in the footwall underground has been observed since the late 1980s.

To accurately forecast the global stability of the footwall at the Kiirunavaara mine with increasing mining depth is essential for the continued operation and design of the infrastructure located both in and on the surface adjacent to the footwall. Even though most of the production critical infrastructure (skip shafts, crushers etc.) is located at a considerable distance from the ore contact a large scale movement/failure in the footwall could drastically impede the mining operations.

The progressive rock mass movement in the footwall is currently indicated by surface subsidence spot observations underground. Underground failure surfaces have traditionally been estimated using empirical relationships.

1.1. The Kiirunavaara mine

The Kiirunavaara mine is a large scale iron ore sublevel caving operation producing 28 Mt (million metric tons) p.a., near the city of Kiruna, Sweden. The main orebody is mined using SLC with the current main haulage level at level 1365 m situated at a depth of roughly 1100 m (actual depth from ground surface).

1.1.1. Coordinate system

In general terms the orientation of the mine is north to south with the footwall on the west side. Orientation and naming of objects and infrastructure is associated to a local 3D coordinate system with vertical z-axis originating at the pre-mining top of the Kiirunavaara mountain. The local y-axis is roughly oriented north to south and follows the general strike of the orebody, the x-axis is oriented roughly west to east. Z-coordinates increases with depths, y-coordinates increases southwards and x-coordinates increases eastwards into and beyond the hangingwall, see Figure 1.
Figure 1 Coordinate orientation of the Kiirunavaara mine

1.1.2. Orebody
The orebody undercuts the hangingwall as it dips roughly 60° east. The orebody is sectioned into production blocks. The blocks are named from their position along the y-coordinate axis combined with the level (z-coordinate) on which the block is/was excavated.

1.1.3. Infrastructure
The LKAB surface and underground infrastructure is located on and within the footwall. The underground infrastructure is situated inside the footwall and aligned parallel to the length of the orebody, see Figure 2. Most of the permanent infrastructure such as crushers, skip shafts and workshops are located far into the footwall in the so-called “centrala anläggningar” (CA), the CA area is located between y-coordinates Y22 and Y25. From the CA-area access and transportation drifts extend eastwards to the actual mining areas.
The infrastructure in the mining areas primarily consists of roads and ore passes designated by their placement according to the local y-coordinate e.g. ore pass group 19 is located roughly at Y19. The ore passes are fed from overlying production areas in the orebody which are divided into blocks designated by their y-coordinates.

The northern part of the mine is designated “Sjömalmen”; excavation of this volume did not originate in the open pit as contrary to the rest of the mine. The orebody has been accessed from the underground infrastructure only.

1.2. Objectives
The objective of this report is to compile a unified description of the collected monitoring data on the footwall.

1.3. Scope
This review concentrates on the measuring and monitoring systems in the Kiirunavaara footwall. The review includes:

- Documentation on currently active as well as decommissioned systems.
- Documented monitoring data from the above systems.
- Earlier numerical models of the footwall.

Outside of the scope of this report are:
- Review of literature covering other mines.
- Development or analysis of prognosis methods.
- Prognosis work for future mining.
- Hangingwall failure and related deformations.
- Seismic data.
2. Surface deformation

Surface deformation is here defined as measurable disturbances on the ground surface caused by the mining activities such as settlements and crack formation. Seasonal disturbances due to changing water conditions and frost heave are thus outside the used definition of deformation.

2.1. Current surface monitoring systems

2.1.1. Surface mapping

The first systematic surface crack mapping was conducted by Lupo in 1992 and additional mapping was performed by Lupo in 1994 and 1995 (Lupo, 1996). These mapping campaigns were initiated due to the observation of surface cracking on the access road to the then decommissioned Zenobia (open pit) crusher. In addition to the mapping results, surface deformation prognoses were presented.

In Lupo’s first map, for 1992, surface cracking was limited to the previously described area around the Zenobia crusher and to the end of the open pit at Y14. These areas were linked by inferred cracking between the points of observation. Two years later, in 1994, the extent of the surface cracking had expanded noticeably and fractures were now observable continuously between the far north part and Y22, plausibly even beyond this point. The area south of Y22 was covered by waste rock deposits but Lupo inferred continued crack extension underneath these deposits from the “stair-stepped” surface features in the waste rock aligned with the fracture lines. The 1995 map showed no significant changes in the extent of the deformed area other than that the fracture line seemed to have moved approximately 5 m westward into the footwall indicating progressive large scale footwall failure. The low number of surface survey points at this time limited the possibilities to exactly determine the extent of the influenced volume on the footwall side (Lupo, 1996).

Surface mapping was conducted again in 2000 and reported by Henry (2001). At the time surface fractures were only documented in the Zenobia area, the progression westward was believed to be limited by large scale structures running sub parallel to the orebody. The eastern parts of these limiting structures had settled several meters (>5 m) in 2001 see Figure 3, (Henry, 2001) and the settlement has since then continuously progressed, see Figure 4 (Mäkitaavola & Sjöberg, 2012).
In the Zenobia area structures of varying size and orientations had opened and the “Kapten” oriented (N-S) structures had predominantly sheared, see Figure 5 (Henry, 2001). The areas where cracks were observed were the same locations as reported by Lupo (1996) (Mäkitaavola & Sjöberg, 2012).
The movements on the surface were stated to be in the order of 25 times larger than correlated movements in discontinuities underground. At the time the bottom of the pit (caved rock mass top) was located on the 300 m level leaving a 220 m high unsupported slope, the material in the free slope was concluded to expand without confinement allowing structures striking south to slide unhindered (Henry, 2001).

Figure 5 Reactivated Kapten oriented structure in Zenobia from (Henry, 2001)

Surface mapping has been performed annually on the footwall since 2008 (Mäkitaavola & Sjöberg, 2012), visible cracks on the surface are mapped in relation to presence and relative orientation to the open pit (photos exist from 2006). Mapping is performed on foot in accessible areas. In the southernmost parts no cracking has yet been observed and mapping is explicitly performed only for Y4 to Y22. Cracking outside the open pit area has so far only been observed for "Sjömalmen" i.e. Y4-Y13 (Mäkitaavola, 2013).

2.1.2. Aerial photographs
Aerial photographs of the open pit area and the footwall are captured on a yearly basis from helicopter. The helicopter over flight and photo capture is performed by an external contractor in one session during a snow free month, typically during early June. Optical mapping of surface features and large scale settlement is also performed during the flight by LKAB research staff (Mäkitaavola, 2013).

2.1.3. GPS measuring lines
Surface deformation monitoring is performed on both the footwall and hangingwall by utilising GPS hubs placed in a number of measuring lines. The footwall measuring lines includes a total of 84 hubs, see Figure 6, of which the majority are measured once a year,. Points of special interest where larger deformations are expected e.g. at the "Sjömalmen" rock cap are measured every quarter. Also hubs in close proximity to existing infrastructure such as roads,
parking areas and office buildings are measured on a quarterly basis (Mäkitaavola & Sjöberg, 2012).

Figure 6 GPS measuring hubs on the footwall adapted from Nilsson (2012)

2.1.4. InSAR

Interferometric Synthetic Aperture Radar (InSAR) is a satellite based movement monitoring system. Radar images are captured periodically over an area and surface movements are calculated from changes between consecutive images. Three types of InSAR analyses are performed at Kiirunavaara; Coherent Target Monitoring (CTM), corner reflectors (CR) and Differential InSAR (DInSAR).

The differences between the methods are the reference objects used, in CTM well defined objects such as roofs and buildings are used. In DInSAR large scale natural formations such as slopes and waste piles are instead used. The arguably best precision is produced by CR, 3 mm in E-W and 14 mm in Z (Mäkitaavola, 2013).

2.1.5. Extensometers

Three borehole extensometers were installed in 2009 to monitor near surface deformations in close proximity to the LKAB office. The extensometer holes were drilled with 50° inclination dipping towards the open pit, see Figure 7, with a hole length of 105 m. Each hole contains two coupled extensometers. The installation orientation was chosen based on “the measured resultant deformation vector at the ground surface, at the time of installation” (Mäkitaavola & Sjöberg, 2012).
2.2. Surface deformation data

2.2.1. Surface mapping

Between 2008 and 2012 new footwall cracks have only been observed in close proximity of the “Sjömalmaren” cap rock, see Figure 9 and Figure 10, at approximately Y4-Y13.
Crack development in Zenobia as described by Lupo (1996) is likely to continue but is not explicitly mapped. The primary interest of the mapping campaign has been to track the outer extent of visible crack formation and the behaviour of cracks which may directly influence the surface infrastructure (Mäkitaavola, 2013).

Figure 8 Location of mapped surface cracks (purple) in 2008 adapted from Mäkitaavola & Sjöberg (2012)
In 2012 the cracks marked in Figure 9 and Figure 10 had coalesced and progressed southward and were last mapped behind hub Z8 (see cut out in Figure 10). Detailed follow up has since 2008 been performed for the outermost (most western) cracks only (Mäkitaavola & Sjöberg, 2012). Mapping results are referenced in more detail by Mäkitaavola & Sjöberg (2012).

2.2.2. GPS

Surface deformation data for the years 2010 (yearly) and 2009 (cumulative) was presented by Mäkitaavola (2010a; 2010b). GPS measurement on a number of well-defined points grouped in measurement lines, see Figure 10, were analysed and discussed briefly. The major conclusions were that the footwall suffers smooth deformations with regard to magnitude, e.g. measuring point “P4” in Figure 10 and Figure 11, and that some of the measuring points should be replaced due to damage and influence from external forces such as vehicle hits (Mäkitaavola, 2010a). Some points showed noticeably accelerated movements from 2009 to 2010, e.g. “Z12” in Figure 10 and Figure 12, the acceleration corresponded well to predictions based on the fracturing of the crown-pillar in the “Sjömalmen” area (Mäkitaavola, 2010b).
Figure 10 Footwall surface GPS measuring points, blue arrow indicates measuring point “P4” and red arrow “Z12”

Figure 11 Movement of measuring point “P4” between 2005 and 2010 from Mäkitaavola (2010a) [X-direction black, Y-direction Yellow, Z-direction blue, Y axis indicate deformation in m]
Figure 12 Movement of measuring point “Z12” between 2008 and 2010 from Mäkitaavola (2010a) [X-direction black, Y-direction Yellow, Z-direction blue, Y axis indicate deformation in m]

2.2.3. \textit{InSAR}
Corner reflectors have so far only been installed on the hangingwall and validation by comparison to GPS data is on-going in a parallel project. Validation of the footwall InSAR data is yet to be performed.
None of the described satellite methods are usable for the purpose of monitoring the open pit bottom due to the satellite ocular incidence angle in combination with the surface texture of the pit bottom (Mäkitaavola, 2013).

2.2.4. \textit{Extensometer}
Extensometer data is continuously collected and analysed by LKAB staff. Only negligible deformations had been recorded at the time of the last report issued 2012 (Mäkitaavola & Sjöberg, 2012).

3. Underground deformations

Underground deformations are defined as measurable disturbances caused by mining activities. Examples of deformation are opening or shearing of pre-existing discontinuities, rock mass volume increase or decrease and relative translations or movement of
excavations. Rock mass volume changes explicitly caused by extraction of material (fall outs, excavation) lies outside the definition of deformations.

3.1. Decommissioned underground deformation monitoring equipment

3.1.1. Distometer lines

A number of distometer lines (wall-to-wall convergence) have been installed in the so-called upper footwall, that is, above the current active mining level. The measurement data have been saved as Excel spread sheets and notations on mine maps in MS (MicroStation) and stored on the LKAB server under "R:\Bergmek\mätningar\distometer". The location of some of the individual measurement lines can be found in Table 1.

Table 1 Location of distometer lines from (Henry, 2001)

<table>
<thead>
<tr>
<th>Level</th>
<th>Line number</th>
<th>Y-coordinate</th>
<th>X-coordinate</th>
<th>Name of MS map file (R:\Bergmek)</th>
</tr>
</thead>
<tbody>
<tr>
<td>230</td>
<td>1 to 13</td>
<td>2175</td>
<td>5750-5845</td>
<td>Bv230.dgn</td>
</tr>
<tr>
<td>V25-386</td>
<td>1 to 3</td>
<td>2100-2132</td>
<td>5908-5933</td>
<td>Bv25370.dgn</td>
</tr>
<tr>
<td>509</td>
<td>1 to 6</td>
<td>2240-2400</td>
<td>5840-5975</td>
<td>Bv25500.dgn</td>
</tr>
<tr>
<td>KP55</td>
<td>1 to 6</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>540</td>
<td>1 to 4</td>
<td>2400</td>
<td>5840-5940</td>
<td>Bv540.dgn</td>
</tr>
</tbody>
</table>

Some of the lines were installed during the mid-80s and were in 2001 already well inside the fractured volume and measuring of these had thus been abandoned. Another 18 lines were installed during April 2000 in the CA drift on level 230 m at X5750 to X5600, the mined out ore contact on this level was then located at X5800 (Henry, 2001).

An evaluation of underground theodolite measurements was presented by Henry (2000). The reliability of the bulk of the measured values was shown to be questionable. Negligible errors were multiplied as each point was measured in reference to the previous point resulting in a significant cumulative error. This became more pronounced for higher measuring point numbers. The few reliable points indicated a movement of the rock mass towards the extraction area. This corresponded to the expected behaviour of elastic expansion due to the removal of the horizontal stress component.

3.1.2. Coaxial cables & Seismic systems

TDR or Time Domain Reflectometry has been utilized since the 1980’s to monitor the rock mass with satisfactory results (Henry & Dahner-Lindkvist, 2000). The TDR technique works by measuring the response of an electrical pulse in a grouted deformable cable. Shearing or extension of the cable will alter the transmittance characteristics along the cable and indicate rock mass movement (Dowding et al., 1988). Due to the early installation many of the installation points are now inside the failure zone and can no longer be accessed. The high cost of installing new TDR cables led to LKAB replacing this system with a macro-seismic system (recording large magnitude events) from CANMET for indicating rock mass movement (Henry & Dahner-Lindkvist, 2000).
The system from CANMET installed in 2000 recorded 235 events during the first 18 months of use, all activity was centred far from/under the production areas but close to the infrastructure below level 698 m (Sandström, 2003). This system was subsequently replaced by a micro-seismic system from IMS in 2003 (Stålnacke, 2012).

3.1.3. **Shear cables (TDR)**
Shear cables has been installed in the upper footwall mainly along road 25 and level 230 m. The location of the cables can be found in Table 2 (Henry, 2001).

**Table 2 Location of shear cables from Henry (2001)**

<table>
<thead>
<tr>
<th>Cable number</th>
<th>Y-coordinate</th>
<th>X-coordinate</th>
<th>Z-coordinate</th>
<th>Name of MS mine map file (R:\Bergmek)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5371</td>
<td>2270</td>
<td>5812</td>
<td>-526</td>
<td>Bv2500.dgn</td>
</tr>
<tr>
<td>5383</td>
<td>2231</td>
<td>6010</td>
<td>-613</td>
<td>Bv25610.dgn</td>
</tr>
<tr>
<td>5384</td>
<td>2335</td>
<td>6015</td>
<td>-625</td>
<td>Bv25610.dgn</td>
</tr>
<tr>
<td>5402</td>
<td>2529</td>
<td>6003</td>
<td>-642</td>
<td>Bv25610.dgn</td>
</tr>
<tr>
<td>5587</td>
<td>2529</td>
<td>6014</td>
<td>-642</td>
<td>Bv25610.dgn</td>
</tr>
<tr>
<td>5372</td>
<td>2379</td>
<td>6007</td>
<td>-661</td>
<td>Bv25650.dgn</td>
</tr>
<tr>
<td>5339</td>
<td>2239</td>
<td>6039</td>
<td>-700</td>
<td>Bv25650.dgn</td>
</tr>
<tr>
<td>5373</td>
<td>2314</td>
<td>6051</td>
<td>-708</td>
<td>Bv25710.dgn</td>
</tr>
<tr>
<td>5374</td>
<td>2389</td>
<td>6030</td>
<td>-715</td>
<td>Bv25710.dgn</td>
</tr>
<tr>
<td>5370</td>
<td>2578</td>
<td>5962</td>
<td>-722</td>
<td>Bv25710.dgn</td>
</tr>
<tr>
<td>5355</td>
<td>2520</td>
<td>6030</td>
<td>-745</td>
<td>Bv25710.dgn</td>
</tr>
<tr>
<td>5411</td>
<td>2143</td>
<td>6001</td>
<td>-784</td>
<td>Bv25760.dgn</td>
</tr>
<tr>
<td>54108</td>
<td>2182</td>
<td>6082</td>
<td>-798</td>
<td>Bv25760.dgn</td>
</tr>
</tbody>
</table>

3.1.4. **Convergence measurements**
Convergence measurements have only been used on a few occasions and in specific locations to monitor the behaviour of individual drifts. The most commonly used measuring device (except for the distometer lines) has been the "grängsesstång" (an extendable roof-floor convergence measuring pole). The most recent found data of this type was recorded between 2002 and 2006 e.g. Krekula (2004).

3.1.5. **Joint displacement measurements**
Movement indicators were installed along one of the major planes in the footwall during 2000 and 2001 according to Table 3 (Krekula, 2004). The devices measures the opening or shearing of a structure at a specified location. The GeoKon devices cannot be optically interpreted but needs to be connected to specific hardware.
Table 3 Movement indicators adapted from Krekula (2004) and Henry (2001) combined with measurement data

<table>
<thead>
<tr>
<th>Number</th>
<th>Installation</th>
<th>Location</th>
<th>Date</th>
<th>Total movement 2004</th>
<th>Movement/year</th>
</tr>
</thead>
<tbody>
<tr>
<td>17874</td>
<td>GeoKon 4420-50 mm- LAH2</td>
<td>Y17, level 320 m</td>
<td>2001-04</td>
<td></td>
<td></td>
</tr>
<tr>
<td>17875</td>
<td>GeoKon- LAH1</td>
<td>Y24, level 320 m</td>
<td>2001-04-02</td>
<td>5.3 mm</td>
<td>1.8 mm</td>
</tr>
<tr>
<td>Ozgaug30-Y24</td>
<td>Ruler, measures vertical and horizontal movements</td>
<td>Y24, level 509 m</td>
<td>2000-06-14</td>
<td>7 mm (combined movement, the largest contributor is vertical)</td>
<td>2.1 mm</td>
</tr>
<tr>
<td>17876</td>
<td>GeoKon- Diagonally over joint -LAH2</td>
<td>Y25, level 509 m</td>
<td>2001-04-02</td>
<td>0.6 mm</td>
<td>0.2 mm</td>
</tr>
<tr>
<td>17877</td>
<td>GeoKon 4420-50 mm -Kapten</td>
<td>Road 25, level 620 m</td>
<td>2001-04</td>
<td></td>
<td></td>
</tr>
<tr>
<td>17878</td>
<td>GeoKon 4420-50 mm -Kapten</td>
<td>Road 25, level 630 m</td>
<td>2001-04</td>
<td></td>
<td></td>
</tr>
<tr>
<td>17879</td>
<td>GeoKon- Diagonally over joint -LAH1</td>
<td>Y25, level 659 m</td>
<td>2001-04-04 Uninstalled</td>
<td>5.6 mm</td>
<td>2.4 mm</td>
</tr>
</tbody>
</table>

3.1.6. **Underground mapping**

Mapping of the progression of the extent of the outer fracturing in the footwall was to be continuously performed every two years according to Hedin (1992), or once a year according to Hansen-Haug (2000). Especially the mid-to-late 1990s and late 2000s seem to suffer gaps in documentation, only notations on mine maps seems to exist. No interpretations or comprehensive descriptions of the failures have been found. Sjöberg (1999) presented mapped failures from 1998 but this mapping campaign does not seem to have been systematically documented at LKAB and has therefore not been accessed in full. Specific mapping campaigns tracking the footwall failure have been performed most recently by Henry (2001), Krekula, (2004) and by the author, Nilsson, (2012b) and Nilsson et al. (2014). The campaigns have been centred on the CA area (Y22-24) and some indications of the current location of the failure surface between these coordinates were given by Nilsson (2012a), see Figure 14. Additional data was presented in Nilsson et al. (2014).
Figure 13 Estimated outer failure line at the CA area from Nilsson (2012a) [Axis denote mine coordinates]

Figure 14 indicates small deviations in the position of the fracture line between the years 2004 and 2012. However, it was at the time difficult to draw conclusions from this graph alone (Nilsson, 2012b) as deviations only start to be noticeable at the deepest of the mapped levels i.e. level 775 m. Henry (2001) stated that “the northern part of the mine (>Y30) is densely populated by a great number of drifts, roads and infrastructure. So far the outer failure line can be easily observed. This is not the case in the mines southern part and there is a risk that this will also be the case in the mines northern part if the instability moves further west, deeper into the footwall. In such a case one would have a place of observation in the CA-drift”.

3.2. Deformation data from decommissioned systems

3.2.1. Irrelevant data
Due to the conclusions by Henry (2000) the theodolite data was considered irrelevant. The convergence measurements using “Grängesstång” indicates only local deformations and gives in the authors opinion data that is too limited to be of any use within the scope of this investigation.
3.2.2. Relevant data

Joint plane measurement data from the devices listed in Table 3 are presented in Figure 14. It is clear displacements were occurring for all measured points but the magnitude differed substantially between the points. The most significant deformations were recorded for devices 17875 and 17879. Device 17879 was uninstalled in 2001 and did thus not produce any more recent data. Device 17875 (Figure 14B) has continued to displace and new measurements in 2014 indicate a displacement rate of 1 mm/year.

![Figure 14 Joint displacement data, see location of measurement in Table 3](image)

3.3. Currently used monitoring equipment

The seismic system is currently the only systematic measuring system monitoring the footwall underground (Mäkitaavola, 2013).
In 2003/2004 the CAMNET system was replaced by a new system from IMS (Stålnacke, 2012). The first array was installed during 2004 in the area around “Sjömalmen” to monitor the blocking of the rock cap. A larger array covering the entire orebody was installed during 2008 with the purpose of monitoring the seismicity in the production areas (Mäkitaavola & Sjöberg, 2012). This system constituted 127 working geophones at the end of 2011. The number of geophones at the end of 2013 was around 220 Stöckel et al. (2013). The current aim is to achieve accuracy able to locate an event within 20 m with a sensitivity limit of -1.5 in the local magnitude (minimum magnitude adequately recorded).

The majority of the geophones were installed in the footwall, see Figure 21, in close proximity to the active mining areas. Few geophones are located on the higher (already excavated) levels, see Figure 22. As most of the geophones are placed near the active mining areas there are possible limitations in monitoring the large scale footwall failure in the upper portions of the mine. The seismic data has thus so far not been incorporated in the deformation prognoses (Mäkitaavola & Sjöberg, 2012).

Figure 15 Top view of the geophone placement in 2011-12-31 with level 1045 m as background
3.4. **Current underground deformation data**
Seismic data is outside the scope of this review and has yet to be evaluated.

4. Geomechanical properties

4.1. **Geological mapping**

4.1.1. **General**
The main host rock type in the footwall is Precambrian aged low quartz syenite porphyry. At the ore boundary the presence of 0.2-0.4 m thick zones of amphibolite and actinolite skarns has been documented (Lupo, 1996). The intact rock strength was referenced by Sjöberg (1999) as varying between 140-300 MPa. The main rock type was in one case referred to as trachyo-andesite (Henry & Dahner-Lindkvist, 2000), which mechanical properties are likely close to syenite porphyry (Sjöberg, 1999). Sandström (2003) specified the ore/footwall contact to include breccia consisting of actinolitic amphibole with a RMR of 22, this contact was specified to be more prominent in the north part of Kiirunavaara. The RMR of the footwall has been investigated by several authors and was referenced in a range of 49 to 68 by Sandström (2003). The porphyry is subsequently replaced by other rock types as one moves further into the footwall, see Figure 23 and e.g. Andersson (2009). The predominant joint orientation close to the production areas in the mine as a whole is the locally denoted “Kapten” orientation striking roughly WNW-ESE and dipping steeply to the south. A second dominant orientation becoming more common as the depth increases is SW-NE dipping steeply to NW, in addition to these sets ore parallel joints striking N-S and dipping steeply to the east are common throughout the mine. In general the joints do not penetrate the ore contact but the major orientations are roughly the same on both sides of the contact (Mattsson, et al. 2010).
The footwall porphyry is subdivided into 5 categories with respect to the strength properties, see Table 4, these categories are LKAB internal denotations and has little or no geological meaning. The different “types” of porphyry are to a large extent intermixed in the rock mass (Mäkitaavola, 2013) and to accurately determine the extent of each subclass in a specified volume is likely to be impractical. In later sections of this report rock mass properties in specified volumes will relate to averaged values derived from the combination of joint properties and relative distributions of the rock type subclasses.

Figure 17 Geological map of the Kiruna area from Mattsson et al. (2010)
Table 4 Rock type denotations adapted from Sandström (2003), Mäkitaavola & Sjöberg (2012) and Mäkitaavola (2013)

<table>
<thead>
<tr>
<th>Type</th>
<th>Rock type</th>
<th>LKAB denotation (Swedish in parenthesis)</th>
<th>Density (Kg/m³)</th>
<th>Young’s modulus (GPa)</th>
<th>Poisson’s ratio</th>
<th>Compressive / tensile strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SP1</td>
<td>Trachyandesite</td>
<td>Syenite porphyry (Syenitporfyr/trakyt)</td>
<td>2800</td>
<td>70</td>
<td>0.20-0.27</td>
<td>300-430/10</td>
</tr>
<tr>
<td>SP2</td>
<td>Trachyandesite</td>
<td>Syenite porphyry (Syenitporfyr/tät omvandlad trakyt)</td>
<td>2800</td>
<td></td>
<td></td>
<td>400/-</td>
</tr>
<tr>
<td>SP3</td>
<td>Trachyandesite with amygdules</td>
<td>Amygdule porphyry (Mandelstensporfyr)</td>
<td>2800</td>
<td>44-60</td>
<td>0.14-0.24</td>
<td>110-210/11</td>
</tr>
<tr>
<td>SP4</td>
<td>Idiomorphic trachyte and porphyry</td>
<td>Idiomorphic trachyte and porphyry (Idiomorfporfyr/trakyt)</td>
<td>2800</td>
<td>80</td>
<td>-</td>
<td>430/12</td>
</tr>
<tr>
<td>SP5</td>
<td>Weathered trachyandesite</td>
<td>Weathered trachyandesite (Vittrad trakyt)</td>
<td>2800</td>
<td>75</td>
<td>-</td>
<td>90/10</td>
</tr>
<tr>
<td>GP</td>
<td>Porphyr dyke</td>
<td>Dyke porphyry (Gångporfyr)</td>
<td>2800</td>
<td>75</td>
<td>-</td>
<td>320/-</td>
</tr>
<tr>
<td></td>
<td>Footwall [Average]</td>
<td></td>
<td>2800</td>
<td>70</td>
<td>0.27</td>
<td>-</td>
</tr>
</tbody>
</table>

Apart from the porphyry other rock types are introduced further away from the ore contact, in the CA area a granite rock dome starts at level 800 m and continues towards the depths explicitly mapped down to 1470 m. The granite is in general cut by a large number of joint sets which differ in orientation and properties from the sets found in the porphyry closer to the ore contact. The GSI of the granite varies between 45 and 70. The areas of high GSI are more prone to rock burst. The mean UCS value for the granite is about 265 MPa (Andersson, 2009).
On level 1045 m SP3 is the predominant rock type in combination with SP4, the other subclasses occur only sporadically and no direct relation to the location of observed damage has been seen Malmgren et al. (2008).

4.2. *Major structural planes*

Three major structural planes were identified and described by Henry (2001). The planes showed signs of shear displacement and were denoted LAH 1 to LAH 3. The descriptions of LAH given by Henry (2001) are briefly summarised in the following subchapter. According to Henry (2001) the field observations at the time showed that LAH 1-3 had captured most of the footwall movement, LAH 2 was activated at a later stage than LAH 1 and LAH 3. In addition LAH 1 and LAH 2 are oriented parallel to the orebody and thus never intersects it. Hence, shear along these planes requires shearing of the rock mass between the structure planes and the ore contact. It is further indicated that this rock mass shearing volume is most likely diffuse allowing the upper part of the volume to move in almost pure structural shear rather than by the rotational behaviour expected if the rock mass shear would have been more discreet. Henry (2001) concludes based on this that in the northern part of the mine the primary failure mode for the upper footwall is structurally controlled complex wedge failure. Failures through the rock mass are considered possible but should according to Henry (2001) be evaluated only when structural failures have been dismissed as implausible.

In 2010 the LKAB database contained 4 mapped major fault planes (not related to LAH 1-3) according to Table 5.

**Table 5 Fault planes in the LKAB database according to (Mattsson, et al. 2010)**

<table>
<thead>
<tr>
<th>Fault</th>
<th>Z-coordinate</th>
<th>X-coordinate</th>
<th>Y-coordinate</th>
<th>Strike</th>
<th>Dip</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>716</td>
<td>5955</td>
<td>970</td>
<td>E-W</td>
<td>Steep</td>
<td>1 and 2 in close proximity</td>
</tr>
<tr>
<td>2</td>
<td>716</td>
<td>5955</td>
<td>970</td>
<td>E-W</td>
<td>Steep</td>
<td>1 and 2 in close proximity</td>
</tr>
<tr>
<td>3</td>
<td>878</td>
<td>6216</td>
<td>2861</td>
<td>WSW-ENE</td>
<td>Dip not documented</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>993</td>
<td>6586</td>
<td>4278</td>
<td>WSW-ENE</td>
<td>Dip not documented</td>
<td></td>
</tr>
</tbody>
</table>

4.2.1. *LAH 1, 2 & 3*

A large scale discontinuity set has been mapped explicitly between Y22 and Y26 on level 230 m and continuously down to level 713 m. The structure set is inferred to a depth exceeding 849 m. This set has been denoted LAH 1. The set is inferred to intersect a sub vertical structure at level 230 m. Shear displace estimates along LAH 1 have been recorded from shearing of half pipes with magnitudes ranging from 20 cm to less than 2 cm. Observations indicates that the magnitude of the shear slip decreases with depth.
A similar structure set to LAH 1 was mapped at Y17 on levels 230 and 320 m. This set was denoted LAH 2. Shearing of the structure set was indicated by shotcrete damage on levels 230 and 320 m. The set likely daylights near the entrance to road 16 on the ground surface. A third set denoted LAH 3 was mapped between Y26 and Y31 on levels 230 to 370 m. The structure was open on the levels (normally displaced) but was not encountered on level 420 m (Henry, 2001).

4.3. Geomechanical mapping by section

In 2010 the LKAB database contained about 35 000 individual joints. In Figure 18 these joints have been plotted in vertical profiles for 100 m wide profiles (along the Y-axis). Changes in joint orientations in relation to depth were not considered and are instead indicated in Figure 19. A noticeable difference in joint orientations becomes apparent in Figure 19. The observed “shift” in orientation is primarily derived from differences in mapped volumes. Levels 600 to 1100 m are primarily mapped in close proximity to the orebody (porphyry) while the data for 1100 and down are mostly mapped in the CA-area (granite) further into the footwall (Mattsson, et al. 2010).

The joint orientations displayed in Figure 18 and Figure 19 are simplifications of the “true” properties brought on by averaging of data collected from considerable volumes. The figures visualize data averaged either in the local Y or Z direction. Both data sets are averaged in the local X direction. The averaging in the X direction might impose a considerable bias as joint orientations may alter significantly as one moves further away from the ore contact (authors opinion), see Figure 20 and e.g. Andersson (2009).
Figure 18 Joint orientation with respect to Y-coordinate from Mattsson et al. (2010)
Figure 19 Joint orientation with respect to depth from Mattsson et al. (2010)
Figure 20 Joint orientation "close to ore" (left), "CA-area" (right), both from level 1365 m from Andersson (2009)

To limit the bias and create more representative numerical models the previous mentioned systematic averaging must be considered. While some simplifications are inevitable, or even necessary due to practical reasons, significant bias could be minimized by reducing the studied volumes.

The availability of data on the main properties of the footwall host rock from e.g. Holmdtedt (1996); Andersson (2009) and Mattsson et al., (2010) are presented as an overview in Table 6.
### Table 6 Availability of the main properties of the footwall host rock

<table>
<thead>
<tr>
<th>Available data</th>
<th>Joint orientation data</th>
<th>c-factor</th>
<th>UCS</th>
<th>GSI</th>
<th>Elastic parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coordinate specific</td>
<td>Y14-45 for &lt;400&lt;Z&lt;1000</td>
<td>Y13-19 Z600-900 Y35-45 Z600-900</td>
<td>Y36 Z1200</td>
<td>Y26, Y33, Y35, Y37 Z1200 Y31,Y34 Z1100-1200 Y32 Z1100 &amp; Z1300</td>
<td>-</td>
</tr>
<tr>
<td>Section specific only</td>
<td>-</td>
<td>-</td>
<td>Y14-45 Z200-700</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>General data</td>
<td>All Y for Z&gt;1100</td>
<td>-</td>
<td>All Y for Z&gt;1100</td>
<td>&quot;Footwall rock&quot;</td>
<td></td>
</tr>
<tr>
<td>No data</td>
<td>Z&lt;400</td>
<td>All sections except Y13-19 Z600-900 Y35-45 Z600-900</td>
<td>Z&gt;800 except Y36 Z1200</td>
<td>Z&lt;1100</td>
<td>-</td>
</tr>
</tbody>
</table>

#### 4.4. Damage mapping

Damage mapping has been performed within this project and is described in detail by the author, Nilsson (2013); Nilsson et al (2014).
5. Previous modelling of the Kiirunavaara footwall

5.1. Overview
The first Numerical models in some way incorporating the Kiirunavaara footwall were presented in the mid-90s. The most relevant of these older models (explicitly investigating the footwall stability) are summarised and described by Nilsson (2012a). The current report focuses on the most recent models by Sjöberg, et al. (2001), Malmgren & Sjöberg (2006), Sjöberg (2007) and Perman & Sjöberg (2011) emphasising the input used in relation to the intended purpose and focus of the models. Numerical input for the caved rock masses were presented by Villegas & Nordlund (2013).

The numerical input data by Sjöberg, et al. (2001) were based both on internal literature from LKAB as well as laboratory data documentation. Based on an argument of the dominance of the subgroup SP1 and the similar strength parameters of SP1 and SP 3 “generally applicable” footwall strength properties were derived according to Table 7.

Table 7 Representative footwall rock mass strength parameters (from Sjöberg, et al. (2001))

<table>
<thead>
<tr>
<th>Denotation</th>
<th>ρ (GPa)</th>
<th>E (GPa)</th>
<th>ν (GPa)</th>
<th>c (MPa)</th>
<th>φ°</th>
<th>Compressive/ tensile strength (MPa)</th>
<th>σcm (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Footwall SP</td>
<td>2800</td>
<td>70</td>
<td>0.27</td>
<td>6.3</td>
<td>53.3</td>
<td>200/1.3</td>
<td>37.8</td>
</tr>
</tbody>
</table>

The values in Table 7 were subsequently used without major modifications (slight variations in cohesion and friction angle) by Malmgren & Sjöberg (2006) and Sjöberg (2007). The values are again found in Perman & Sjöberg (2011) but this time not as the representative “base case” but corresponding to a lower limit value referred to in the original report as “Low 1”. In Perman & Sjöberg (2011) the presented “typical values”, see Table 8, can in turn be backtracked as “high” limiting values in Malmgren & Sjöberg (2006).

Table 8 Representative footwall rock mass strength parameters (from Malmgren & Sjöberg (2006) and Perman & Sjöberg (2011))

<table>
<thead>
<tr>
<th>Denotation</th>
<th>ρ (GPa)</th>
<th>E (GPa)</th>
<th>ν (GPa)</th>
<th>c (MPa)</th>
<th>φ°</th>
<th>Compressive/ tensile strength (MPa)</th>
<th>σcm (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Footwall</td>
<td>2800</td>
<td>70</td>
<td>0.27</td>
<td>10.2</td>
<td>55.1</td>
<td>300/2.23</td>
<td>74.6</td>
</tr>
</tbody>
</table>

One can thus conclude that the numerical analyses related to the footwall during the last decade have all used the same sets of data, all originally compiled and presented by Sjöberg et al. (2001) and that the footwall rock mass consistently has been treated as a single coherent geological unit. Mohr-Coulomb perfectly plastic material models have been used throughout.
5.2. Review

The work by Sjöberg, et al. (2001) was aimed at analysing the stability of ore passes located in the footwall. A global/local modelling scheme was adopted where the global model was used to calculate input stresses for the local stability models. The global models were of continuum type constructed in the Itasca code FLAC. The mine geometry was simplified in a 2D section using an ore width of 80 m and dip of 60°, see Figure 27.

![Figure 21 Principal global model setup used by Sjöberg, et al. (2001)](image)

The global models were evaluated in two stages using an element size of 10 m and 5 m respectively using both elastic and plastic input. The effect of the caved rock masses was examined briefly in the report. The inclusion was shown to influence the secondary stresses, especially close to the excavation front and footwall “surface” but showed insignificant influence on the secondary stress state in the area of the active ore passes which was the focus of the work. It was thus concluded that for the purpose of the work the effect of the caved masses could be neglected, the influence on global footwall stability was not explicitly analysed. The final global models were run without caved rock masses using elastic input shown in Table 7. This choice was based on the argument that inclusion of the plastic input did not significantly affect the areas emphasised in the work.

The work of Malmgren & Sjöberg (2006) focused on the stability of infrastructure on level 1365 m. Analyses were performed mainly with respect to stresses and structurally controlled failure (wedge formation). The stress analyses were performed using a global/local modelling scheme. The models were continuum based and constructed using the Itasca codes FLAC and FLAC3D. The global model was used to generate stress input to the local models aimed at evaluating stability.

In the global model two simplified ore geometries were analysed. In both model setups the ore body dip was set to 55° while the width was set to either 80 m or 160 m, see Figure 28.
Figure 22 Principal global model setup used by Malmgren & Sjöberg (2006)

The models were primarily run in FLAC with an element size of 5 m and the caved rock masses were modelled as a void. The geomechanical input differed slightly from Table 7 as the cohesion was set to 6.67 MPa and the internal friction angle to 52.9°. The applied primary stresses differed somewhat to those used by previous researchers, mainly in respect to the ore parallel stress (minor horizontal stress) in correspondence to the stresses presented by Sandström (2003).

A second set of global models (not used to generate stress input to local models) were constructed in 3D using FLAC3D, the same geomechanical input was used as for the 2D models. Comparison of the derived secondary stresses showed that the 2D assumption was valid for roughly 2 thirds of the length of the orebody, at the edges of the ore body the difference in resulting stress was in the order of 25% with lower values in 3D.

The local models were run with an additional set of input parameters corresponding to Table 8 (but using an internal friction angle of 58.2°) as well as the input used in the global models. This second set of parameters were denoted as “high” values but were stated to indicate better agreement with observed damage in blocks 23 and 35 when evaluating yielded zones. For the wedge analyses the Rocscience software Unwedge was used, input parameters where the generally predominant major joint sets with friction angles referenced to 31-38° (35° was used in the analyses) (Malmgren & Sjöberg, 2006).

Sjöberg (2007) performed sensitivity analyses on ore geometry and model excavation procedures. Two sets of 2D global models with ore body dips of 60° and ore widths of 80 m or 160 m respectively were run to study two different model excavation procedures. Yielded zones were compared between models either excavated a sublevel “bit-by-bit” with “actual” strength parameters applied or excavating the entire sublevel “in-one-go” using high strength
parameters and after an equilibrium run applying the “actual strength”. The caved rock masses were modelled as a void, the “actual” rock strength parameters applied were the same as those in Table 7. No actual calibration to observations was made explicitly within the scope of the report. Sjöberg (2007) concluded that the modelled ore width significantly affected the secondary stresses and yielded zones while excavation procedure did not. Perman & Sjöberg (2011) used a 3D 3DEC model to analyse seismic moment primarily in block 19. The model setup used for the mine and the ore body is shown in Figure 29. The caved rock masses were modelled as a low stiffness material.

Figure 23 Block model used by Perman & Sjöberg (2011)

The geomechanical input used as base case were according to Table 8. The parameters used by Malmgren & Sjöberg (2006) were also evaluated but the yielded zones did not reach the areas of interest for either parameter combination. It was concluded that the base case parameters were neither better nor worse than the alternatives and for the purpose of the specific investigation the base case parameters could be applied. Large scale structures judged to be seismically active were also included for block 19 as continuous planes, see Figure 31. The geomechanical properties of these planes were \( k_n \) (joint normal stiffness) = 110 GPa/m, \( k_s \) (joint shear stiffness) = 9 GPa/m with friction angles between 25° and 45° with 5° steps using 30° as base case. The discontinuity tensile strength was set to zero and the cohesion was set to either zero or 0.1 MPa.
Figure 24 Seismically active structures in block 19 incorporated by Perman & Sjöberg (2011)

The 3D model was compared to the 2D models by Malmgren & Sjöberg (2006) as well as to a 3D model in a report not referenced here. It was concluded that using the geomechanical parameters from Table 8 gave good correspondence to the earlier models with respect to both stress distribution and yielded zones. The new 3D model also indicated that the deformations varied in a relatively high degree along the length of the ore body, producing larger deformations where the ore body was less wide. This was stated to likely be caused by stress concentrations in these volumes due to the geometry. The influence on the secondary stress from the ore geometry had also previously been shown by Sjöberg (2007). Perman & Sjöberg (2011) concluded that the choice of rock mass strength parameters was not adequately validated within the project. This was mainly because of insufficient field observations of initiating and fully developed failures within the investigated volumes.
6. Conclusions

The most important findings and conclusions of this review are:

- There have been several systems utilised to monitor the rock mass movement in the footwall underground on both local and larger scale. These systems have all but one fallen out of regular use. Only a seismic monitoring system constituting permanently installed geophones near the excavation level remains active.
- The surface subsidence and the formation of surface cracks are continuously monitored by GPS and manual mapping respectively.
- Three major discontinuities named LAH 1, 2 and 3 could according to a previous author significantly affect the footwall behaviour.
- The numerical analyses related to the footwall during the last decade have all used the same sets of data, all originally compiled and presented in 2001 and that the footwall rock mass consistently has been treated as a single coherent geological unit. Mohr-Coulomb perfectly plastic material models have been used throughout.

References


Nilsson, M. (2013). Damage mapping with respect to the outer fracture surface. Luleå University of Technology: To be published.


